Fatigue Strength of Shear Connectors

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An experimental investigation was undertaken at Lehigh University to determine the fatigue strength of shear connectors for steel and concrete composite beams. Factorial experiments were designed to provide information regarding the effect of stress range and minimum stress level on the cycle life.

Included are fatigue tests of 35 push-out specimens having the concrete slab connected to the steel beam section by ¾-in. stud connectors, 9 fatigue tests of push-out specimens using ⅞-in. stud connectors and 12 fatigue tests of push-out specimens using 4-in., 5.4-lb channel connectors. The test data are described by mathematical equations which express the fatigue life as a function of the stress range.

Based on the reported fatigue tests and previous static and fatigue studies, a design criteria is proposed for the shear connectors of composite beams.

•COMPOSITE construction consisting of a concrete slab attached to steel beams by mechanical shear connectors is widely used for bridge spans of various lengths. Recent static (1) and fatigue studies (2, 3) of composite members have indicated that the currently used design procedure (4) for composite bridge beams is conservative. The wide use of this type of construction would indicate that worthwhile savings could be achieved by a better utilization of the connecting material.

The present design procedure for shear connectors is primarily based on static considerations (5). The useful capacity of connectors was derived from static tests of beams with shear connectors and from push-out specimens by limiting the magnitude of slip to a value which would preclude the yielding of connectors. Design values are obtained by dividing the useful capacity by a suitable factor of safety which insures that the ultimate strength of the member can be developed prior to yielding of connectors. Resulting designs were compared with available fatigue test results which indicated that fatigue failure was not a critical factor in the design. Since fatigue failure of connectors was not possible when this procedure was used, the spacing of connectors was determined from static load considerations. This results in a variable spacing of connectors which is proportional to the ordinate of the shear diagram.

Recent static studies have provided an approach for designing shear connectors so that the flexural strength of the member can be developed without requiring a limitation on the magnitude of slip or preventing yielding of the shear connectors (1). The investigation revealed that the number of connectors required to develop the ultimate strength of a member could be reduced considerably when compared with the requirements in the AASHO specifications. Also, the study showed that connectors need not be spaced in accordance with the intensity of static shear to develop the ultimate strength. Uniform spacing of connectors was satisfactory for most loading conditions and neither ultimate strength nor deflections were appreciably influenced by the uniform spacing of connectors.

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If the shear connector requirements are reduced by decreasing the factor of safety, fatigue failure of connectors may become the governing factor. Fatigue tests of composite beams at Lehigh University (2) and the University of Texas (3) revealed that no direct relationship exists between the static strength and the fatigue strength of connectors. Therefore, it is not advisable to retain the present design procedure and simply reduce the shear connector requirements. The test programs also indicated that when the number of shear connectors was adequate to prevent fatigue failure of connectors, the loss of interaction between slab and beam was not sufficient to cause appreciable increases in stresses and deflections in the beam. The initial fatigue studies did not provide complete information on the fatigue strength of connectors nor the effect of other variables on the fatigue strength.

The purpose of this investigation was to determine the fatigue characteristics of mechanical connectors for composite steel and concrete construction. Previous fatigue tests of composite beams had indicated that considerable variation could be expected in beam specimens, because it was difficult to assess the fatigue damage (2, 3, 6). The failure of one or two connectors could not always be detected and did not significantly affect the beam behavior as the shear was redistributed to other connectors. Also, in beam tests it was not feasible to determine the fatigue behavior of connectors subjected to stress reversal.

Pilot studies indicated that push-out specimens yielded results comparable to beam tests, so this type specimen was selected for the study. A push-out specimen had added advantages in that the loads to which the connectors were subjected could be more easily evaluated because redistribution was not significant. Also, a relatively large number of specimens could be tested more economically using push-out specimens.

EXPERIMENTAL STUDY OF ¾-INCH STUD CONNECTORS

The principal phase of the investigation involved push-out tests of ¾-in. stud connectors. The fatigue characteristics were evaluated by tests of 35 push-out specimens. Twenty-seven of these specimens formed the main experiment, 2 were pilot tests and 6 specimens were added to supplement the data of the main experiment. Each specimen consisted of a 20 by 26¾ by 6-in. reinforced-concrete slab attached by four ¾ by 4-in. stud connectors to an 8WF40 beam section as illustrated in Figure 1. The studs were attached to the ASTM A 36 steel beam sections by a local fabricator. The studs were inspected for soundness following the procedure outlined in a draft of "Recommendations for Materials and for Welding for Steel Channel, Spiral, and Stud Shear Connectors,"

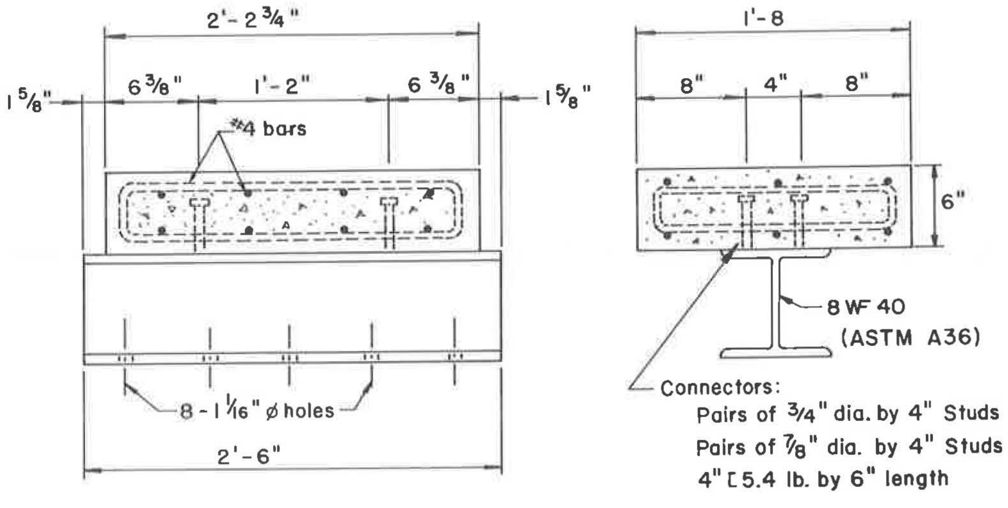


Figure 1. Details of test specimen.

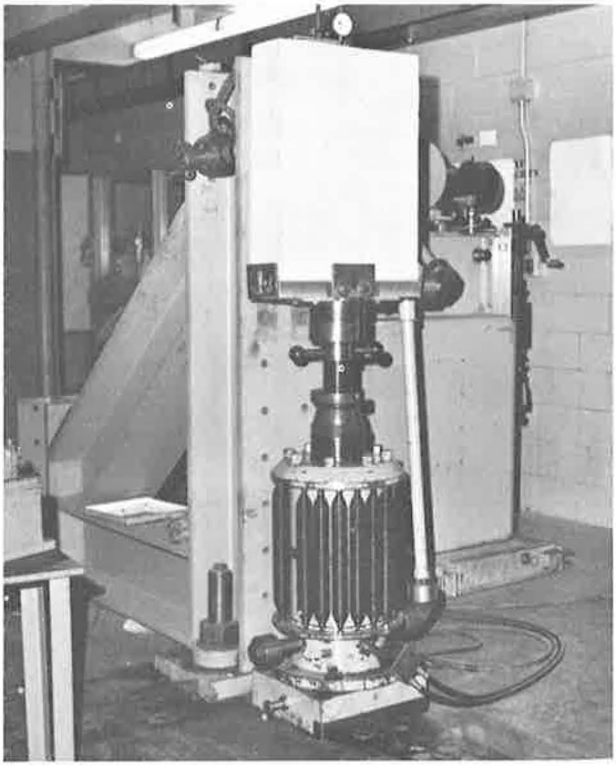


Figure 2. Test setup for loading in one direction.

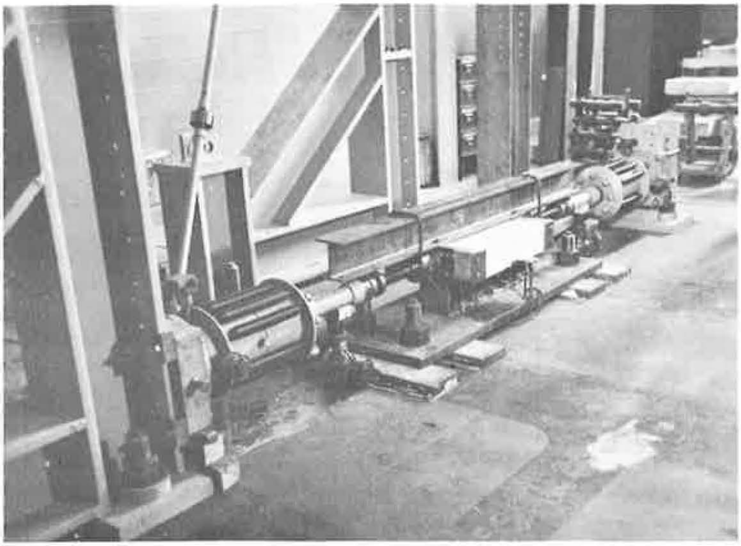


Figure 3. Test setup for stress reversal tests.

proposed by Subcommittee I of the ASCE-ACI Committee on Composite Construction and dated July 10, 1964. The push-out specimens were tested by applying load to the edge of the reinforced-concrete slab as shown in Figure 2. For stress reversal, load was applied to 2 edges of the slab (Fig. 3).

The specimens for the main experiment were cast in groups of 10. All slabs were cast in a horizontal position as in a normal bridge structure. The same concrete mix

TABLE 1

OUTLINE OF ¾-INCH STUD CONNECTOR EXPERIMENT

MAIN EXPERIMENT

Minimum Stress (ksi) \ Maximum Stress (ksi)	10	14	18	22	26
- 6	a 1 A b 1 A c 1 A	a 2 A b 2 A c 2 A	a 3 A b 3 A c 3 A		
2		a 2 B b 2 B c 2 B	a 3 B b 3 B c 3 B	a 4 B b 4 B c 4 B	
10			a 3 C b 3 C c 3 C	a 4 C b 4 C c 4 C	a 5 C b 5 C c 5 C

Minimum Stress (ksi) \ Stress Range (ksi)	8	12	16	20	24
- 6			a 1 A b 1 A c 1 A	a 2 A b 2 A c 2 A	a 3 A b 3 A c 3 A
2		a 2 B b 2 B c 2 B	a 3 B b 3 B c 3 B	a 4 B b 4 B c 4 B	
10	a 3 C b 3 C c 3 C	a 4 C b 4 C c 4 C	a 5 C b 5 C c 5 C		

SUPPLEMENTAL EXPERIMENT

Minimum Stress (ksi) \ Maximum Stress (ksi)	12	20
2	a 6 B b 6 B c 6 B	
10		a 6 C b 6 C c 6 C

Minimum Stress (ksi) \ Stress Range (ksi)	10
2	a 6 B b 6 B c 6 B
10	a 6 C b 6 C c 6 C

proportions were used for each casting. Two cylinders were tested at the beginning of each fatigue test. The specimens were 28 to 92 days old at the time of testing. The mean compressive strength of all cyclinders was 4300 psi, and the standard deviation of the concrete strength was 335 psi.

The tests were conducted with an Amsler hydraulic pulsator and jacks at the loading rates of 250 or 500 cycles per min. The rate of application of load was dependent on the specimen response. The average shear stress on the studs caused by the applied load was computed on the basis of the nominal cross-sectional area of the studs. Stress range is defined as the maximum horizontal shear stress minus the minimum horizontal shear stress in ksi on the cross-sectional area of studs or kips per inch of channel connector.

The main experiment was designed to evaluate 2 controlled variables—the stress range and the minimum stress. An outline of the main experiment design is given in Table 1. Five levels of maximum stress and stress range and 3 levels of minimum stress were selected on the basis of the previous beam experiments in order to establish the fatigue characteristics of the connectors for conditions that exist in bridge structures. Each minimum stress level was combined with 3 levels of maximum stress and stress range in such a manner that 2 complete 2 by 2 factorial experiments were included to obtain data on the effect of minimum stress on the maximum stress and minimum stress on the stress range. These four 2 by 2 factorial experiments are outlined by the dotted lines in Table 1. Three specimens were tested for each combination to provide replication.

Stress levels were assigned to the 27 specimens of the experiment at random and the specimens were assigned to 3 test blocks (a, b, c) as indicated in Table 1. Within each test block of the 2- and 10-ksi minimum stress levels a random order of testing was followed. All 3 test blocks of the reversal specimens (-6 ksi minimum stress) were randomized since a separate test setup was necessary. The random order of testing was followed to prevent variations caused by the controlled variables from being confounded with systematic variations due to uncontrolled variables such as the age of the specimens, behavior of testing equipment, etc.

The results of the main experiment indicated that range of stress rather than maximum stress was the more important variable. A stress range of 10 ksi appeared to be a suitable value for design. In an effort to obtain more data to supplement the main experiment, 6 additional specimens were tested with a stress range of 10 ksi and minimum stress levels of 2 and 10 ksi. This supplemental experiment is also shown in Table 1. The specimens for the supplemental tests were cast in one group. One cylinder was tested at the beginning of each fatigue test. The age of these specimens at the start of testing varied from 55 to 86 days. The mean compressive strength of all cylinders for this series was 3320 psi, and the standard deviation of the concrete strength was 110 psi. Hence, the additional test specimens also provided test data to help ascertain the influence of concrete strength on the fatigue life. In addition, 2 pilot tests are reported which were conducted to aid in the experiment design. The total number of push-out tests of $^3/_4$-in. diameter studs with minimum stress and range of stress as the major variables was 35.

EXPERIMENTAL STUDY OF $^7/_8$-INCH STUD CONNECTORS AND 4-INCH, 5.4-POUND CHANNEL CONNECTORS

The fatigue characteristics of $^7/_8$-in. stud connectors were evaluated by push-out tests identical to those for the $^3/_4$-in. stud connectors. Nine push-out specimens were designed and fabricated similar to the specimen shown in Figure 1. The specimens were tested in the same manner as the $^3/_4$-in. stud connectors.

The 9 test specimens for the factorial experiment were all cast at one time. One cylinder was tested at the beginning of each fatigue test and yielded a mean compressive strength of 4470 psi; the standard deviation was 80 psi. The age of the specimens at the start of testing varied from 53 to 63 days.

The experiment design was identical to that for the $^3/_4$-in. stud connectors except that only one test was made for each combination of stress conditions. An outline of the experiment design is given in Table 2.

TABLE 2

OUTLINE OF 7/8-INCH STUD CONNECTOR EXPERIMENT

Minimum Stress (ksi) / Maximum Stress (kis)	10	14	18	22	26
- 6	e 1 G	e 2 G	e 3 G		
2		e 2 H	e 3 H	e 4 H	
10			e 3 I	e 4 I	e 5 I

Minimum Stress (ksi) / Maximum Stress (ksi)	8	12	16	20	24
- 6			e 1 G	e 2 G	e 3 G
2		e 2 H	e 3 H	e 4 H	
10	e 3 I	e 4 I	e 5 I		

TABLE 3

OUTLINE OF 4-INCH, 5.4-POUND CHANNEL CONNECTOR EXPERIMENT

Minimum Stress (kips/in.) / Maximum Stress (kips/in.)	3.0	3.5	4.0	4.5	5.0
-0.5	d 1 D	d 2 D	d 3 D		
+0.5		d 2 E	d 3 E	d 4 E	
+1.5			d 3 F	d 4 F	d 5 F

Minimum Stress (kips/in.) / Stress Range (kips/in.)	2.5	3.0	3.5	4.0	4.5
-0.5			d 1 D	d 2 D	d 3 D
+0.5		d 2 E	d 3 E	d 4 E	
+1.5	d 3 F	d 4 F	d 5 F		

The fatigue characteristics of 4-in., 4.5-lb channel connectors was evaluated by tests of 12 push-out specimens. Nine of these specimens were part of the factorial experiment and 3 were pilot tests. Each specimen consisted of a reinforced concrete slab identical to that used for the stud connector specimens attached to the 8WF40 steel beam section by two 6-in. lengths of 4-in., 5.4-lb channels. One pilot test had the slab attached to the steel beam by only one 6-in. length of channel. Each channel was attached to the steel beam section by $^3/_{16}$-in. fillet welds placed along the length of the heel and toe. The specimens were tested in the same manner as the $^3/_4$-in. and $^7/_8$-in. stud shear connectors.

The 9 test specimens for the factorial experiment were all cast at one time. One cylinder was tested at the beginning of each fatigue test and yielded a mean compressive strength of 6045 psi, with a standard deviation of 80 psi. The age of the specimens varied from 28 to 76 days.

The tests were all conducted with the Amsler hydraulic pulsator and jacks at the rate of 250 cycles per min. The average force per inch of channel was computed by dividing the applied load by the total channel length. The load was applied to the test specimens of the main experiment by loading the edge of the concrete slab adjacent to the back face of the channel, since this is the orientation commonly used on construction. The load was applied to the opposite edge of the slab during the pilot studies.

The experiment design was the same as for $^7/_8$-in. diameter stud connectors; the outline of the main experiment is given in Table 3.

TEST RESULTS

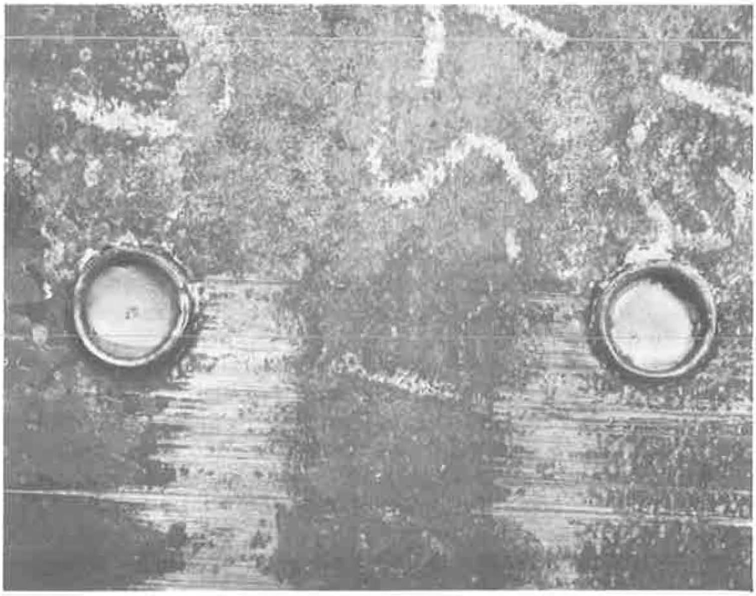
All specimens were tested until failure occurred. For the stud shear connectors 2 different types of failure were apparent. Most of the fatigue failures were initiated at the reinforcement of the stud weld and penetrated into the beam flange causing a concave depression into the beam flange. In a few cases, the fatigue failure initiated at the reinforcement and penetrated through the weld. This latter condition was generally observed to occur when the weld penetration was incomplete. These typical failures are shown in Figure 4. Figure 4(a) shows failure through the weld. The concave depression into the beam flange is apparent in Figure 4(b). The crystalline texture of a typical fatigue fracture is readily apparent. The mode of failure was not a significant variable in these tests.

It was also apparent in the stud connector tests that 2 overall failure modes were evident for the push-out specimens. For the higher stress ranges and the lower minimum stress levels, the 2 studs nearest the applied load failed in fatigue. The remaining 2 studs were usually sheared off by the applied load as their ultimate static strength was exceeded before the machine could be stopped. For the lower stress range and higher stress levels, the applied load was more evenly distributed among the 4 studs and fatigue failures were evident in all 4 connectors.

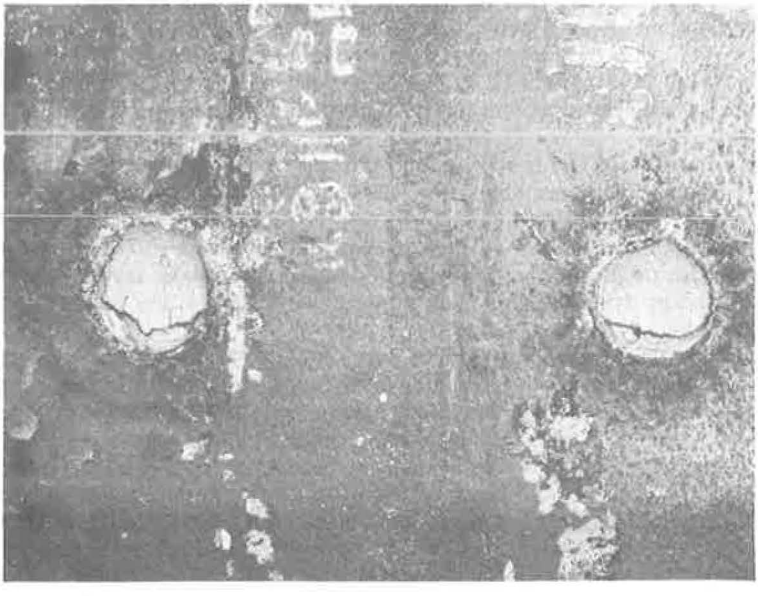
For the channel shear connectors, the fatigue failure was generally initiated in one of the transverse fillet welds and propagated through the weld. In one instance, failure occurred in the channel web. No apparent stress raiser was noted for this case. With the channel connectors it was obvious that the channel nearest the applied load was carrying more load because the fatigue failure always initiated in this connector. The remaining channel was then pulled out of the slab as the static strength was exceeded. Figure 5 shows typical fatigue fractures of the channel connector nearest the applied load and shows the remaining connector that was pushed from the slab as its static strength was exceeded before the machine could be stopped. The specimen on the left is from the main experiment while the specimen on the right is a pilot specimen.

Because of this observed behavior, an additional pilot specimen was fabricated which had only one channel connecting the concrete slab to the steel beam section. No significant difference was observed in the cycle life between the one- or two-channel connector push-out specimens.

The experimental results for the $^3/_4$-in. stud connectors are summarized in Figure 6 in which the stress range is given as a function of the logarithm of the number of cycles to failure for each minimum stress level. The test data from the main experiment are



(a)



(b)

Figure 4. Typical failures of $\frac{3}{4}$-in. diameter studs: (a) failure through the weld; (b) failure penetrating into beam flange.

Figure 5. Failures of channel connectors.

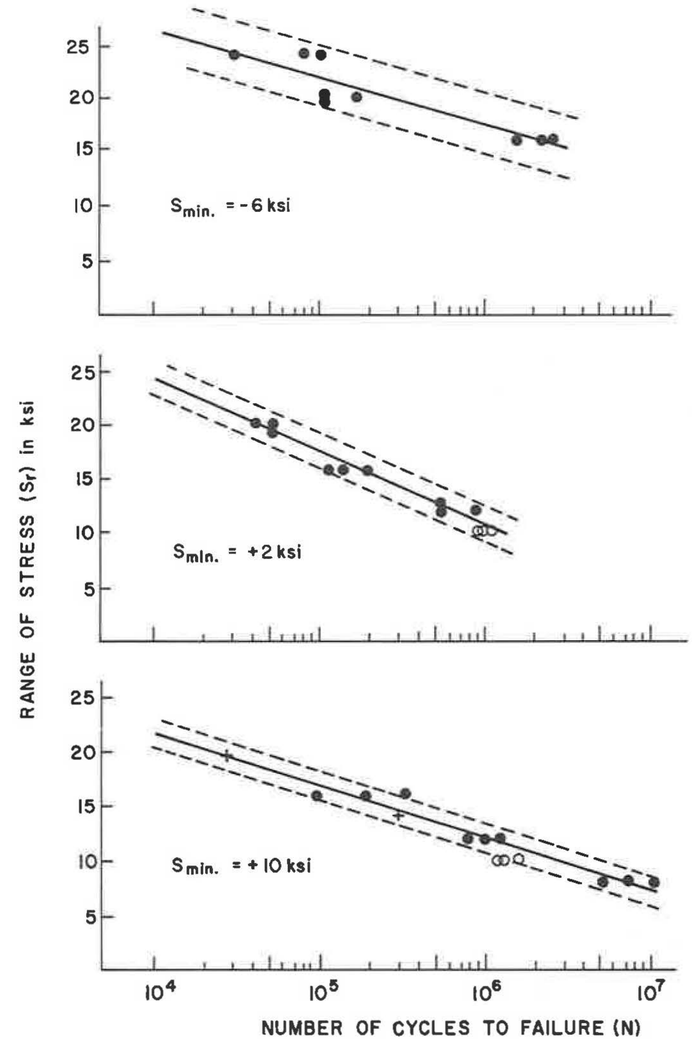


Figure 6. S-N curves for specimens with ¾-in. diameter studs.

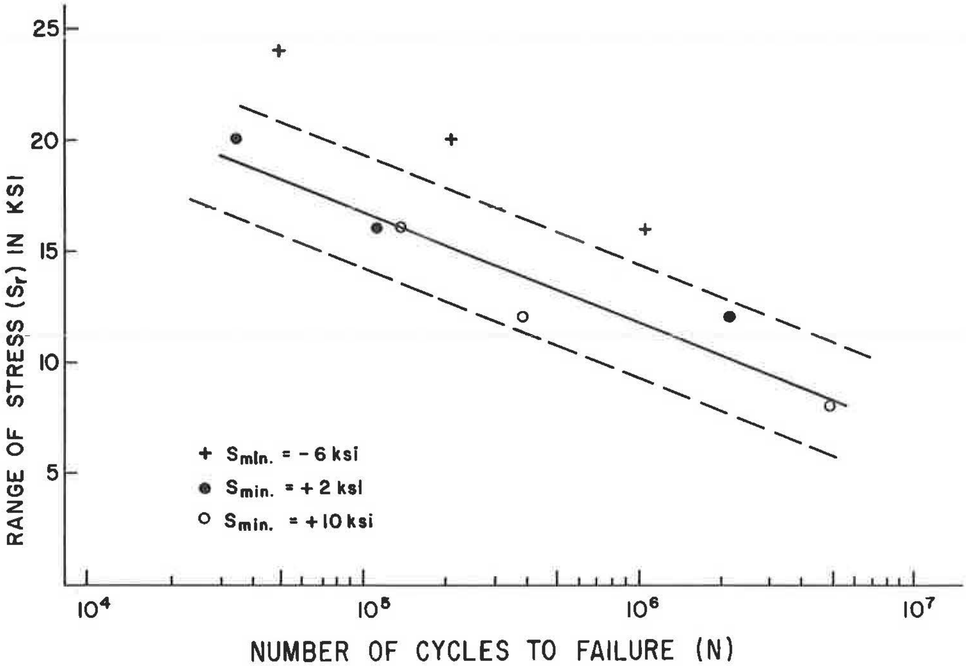


Figure 7. S-N curve for specimens with $\frac{7}{8}$-in. diameter studs.

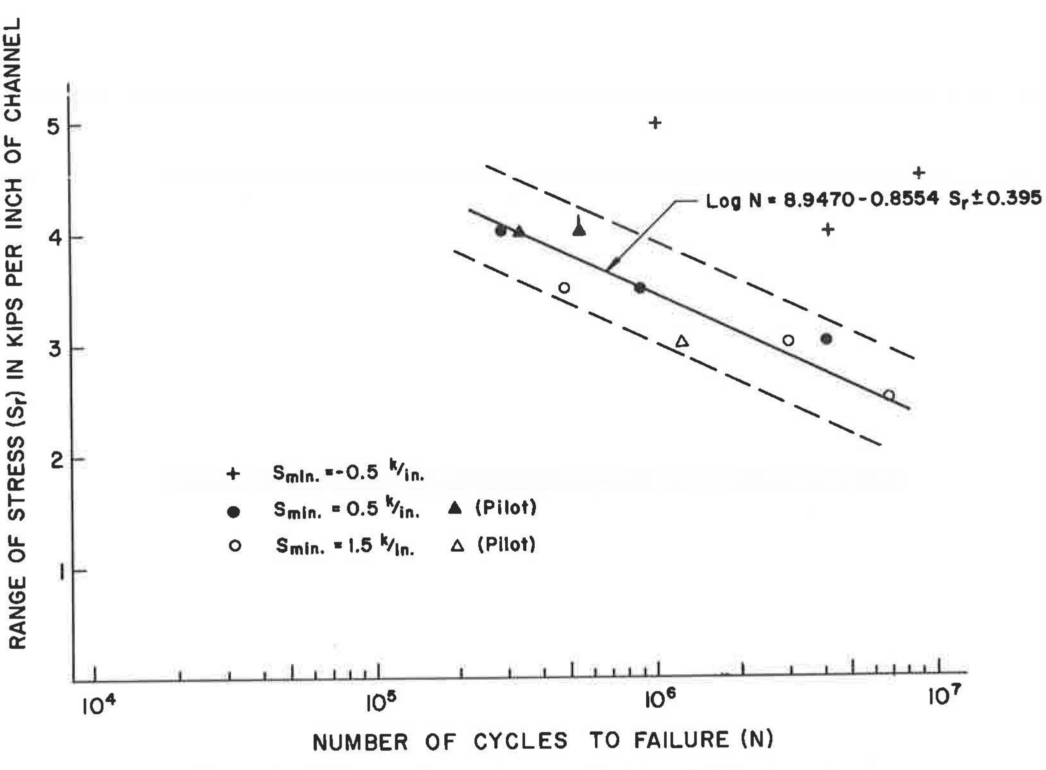


Figure 8. S-N curve for specimens with 4-in., 5.4-lb channels.

plotted as dots. The test data for the supplemental tests are plotted as circles and the pilot tests are plotted as crosses. The cycle life ranged from 27,000 up to 10,275,000 cycles.

The experimental results for the ⅞-in. stud shear connectors are summarized in Figure 7. The test data for S_{min} = -6 ksi are plotted as crosses, the data for S_{min} = +2 ksi are plotted as dots and the data for S_{min} = +10 ksi are plotted as circles. The cycle life ranged from 33,000 to 4,885,100 cycles for the ⅞-in. stud shear connectors. The experimental results for the 4-in., 5.4-lb channel shear connectors are summarized in Figure 8. The test data for S_{min} = -0.5 kips per in. are plotted as crosses, the data for S_{min} = +0.5 kips per in. are plotted as dots and the data for S_{min} = +1.5 kips per in. are plotted as circles. The cycle life ranged from 291,200 to 9,556,300 cycles for the channel shear connectors. The 3 pilot specimens of 4-in., 5.4-lb channel shear connectors are plotted as triangles. The specimen having the single channel connector has a vertical line attached above the triangle. It is obvious that the fatigue strength of the single channel specimen was equivalent to specimens with 2 channel shear connectors. Also, the orientation of the channel connector whether facing toward, as in the pilot tests, or away from the applied load, as in the main tests, had no significant influence on the fatigue life.

ANALYSIS OF TEST RESULTS

Earlier studies (2, 3) had indicated that the fatigue strength of the stud connectors could be represented by a mathematical model of the form

$$\log N = A + B S_r \tag{1}$$

where S_r is the range of shear stress, N the number of cycles to failure and A and B empirical constants. The results of the beam tests are summarized in Figure 9 where circles represent data for ½-in. diameter studs and dots represent data for ¾-in.

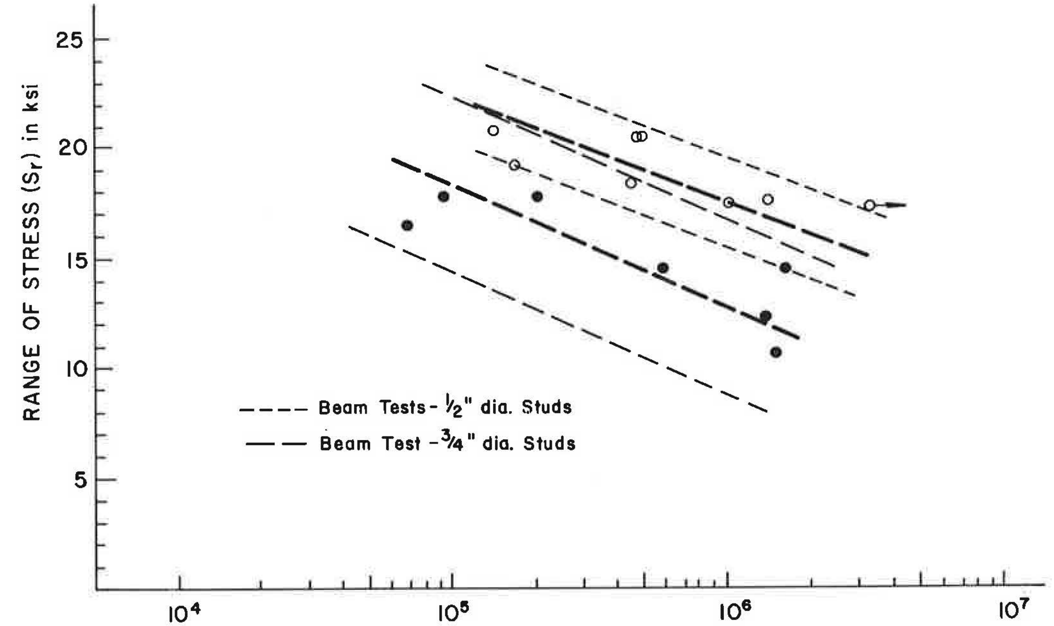


Figure 9. S-N curves for initial connector failure in beams with ½-in. and ¾-in. diameter studs.

diameter studs. All specimens were tested with a low minimum stress level. No apparent leveling off of the S-N curves was noted in the beam tests. The mean regression lines in Figure 9 were developed from data reported earlier (2, 3). Also shown are the limits of dispersion of the test data. The failure criterion for the beam tests was taken as the initial fatigue fracture of the connectors. These tests indicated that ¾-in. stud connectors failed in fatigue at a lower stress than ½-in. stud connectors.

The factorial nature of the current experiments made possible independent determinations of the relative significance of the stress range and the minimum stress level. The analysis of the test data for ¾-in. stud shear connectors showed that the slopes of the S-N curves for the 3 minimum stress levels were not significantly different even at the 10 percent level. This indicated that the stress range affected the cycle life at each minimum stress level to the same degree. On the other hand, the analysis of variance indicated that the distances between the regression lines shown in Figure 6 were significantly different even at the 1 percent level—i.e., the minimum stress was a significant parameter. Hence, stress range and minimum stress accounted for the variations in the experiment.

An examination of the test data for the main experiment and the test data for the supplemental tests indicates that the strength of concrete had only a minor effect on cycle life. The supplemental test specimens with a mean concrete strength of 3320 psi were near the lower limit of dispersion of the test data for specimens with a mean concrete strength of 4300 psi, as can be seen in Figure 6. This was in agreement with the earlier fatigue tests of beam and push-out specimens which had concrete strength varying from about 3000 to 6000 psi (2, 3, 7, 8).

A further evaluation of the test data showed that the reason minimum stress had a significant effect on the cycle life was due to the stress reversal data. When all 3 curves are examined in Figure 6 it is apparent that the stress reversal curve is some distance above the other 2 levels of minimum stress. In fact, an analysis of variance of the test data for minimum stress levels of 2 and 10 ksi indicated that there was no significant difference in the test data and that minimum stress was not significant even at the 10 percent level.

Since the stress reversal specimens had significantly longer fatigue lives for the same stress range than the test data for 2- and 10-ksi minimum stress levels, it was concluded that a conservative estimate of the fatigue life could be obtained for all minimum stress levels by considering only the 2-ksi and 10-ksi minimum stress levels in the analysis. A regression analysis of the test data yielded

$$\log N = 8.072 - 0.1753\, S_r \tag{2}$$

where

S_r = range of shear stress in ksi, $S_{max} - S_{min}$, and
N = number of cycles to failure.

The coefficient of correlation was 0.9323 and the standard error of estimate was 0.1940. The "goodness of the fit" may be judged from Figure 10 where the test data are compared with Eq. 2 shown as the solid line. The equation appears applicable for cycle lives which vary from 10^4 to 10^7. Equation 2 was developed by neglecting the stress reversal data. The limits of dispersion were taken as twice the standard error of estimate and are shown as 2 dashed lines parallel to the regression line. It is readily apparent that such an analysis will provide a greater margin of safety for the stress reversal case. This is not considered to be critical since most connectors will be subjected to a shear loading in only one direction. Also, when shrinkage of the concrete slab occurs, connectors designed for stress reversal may in fact be subjected to such a shear loading.

The results of the tests of ⅞-in. stud shear connectors were summarized in Figure 7. An examination of the test data shows that the ⅞-in. stud connectors behaved similarly to the ¾-in. stud connectors. Figure 11 compares the test data for the ¾-in. and ⅞-in. stud connectors. The data for ¾-in. connectors are shown as dots and for ⅞-in.

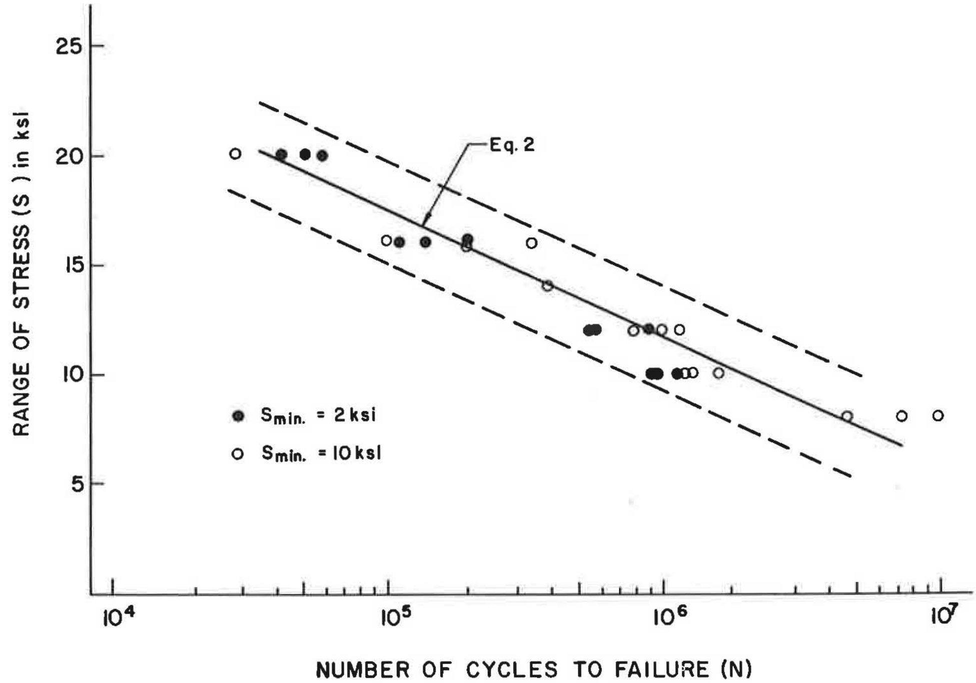


Figure 10. Regression curve for ¾-in. diameter studs.

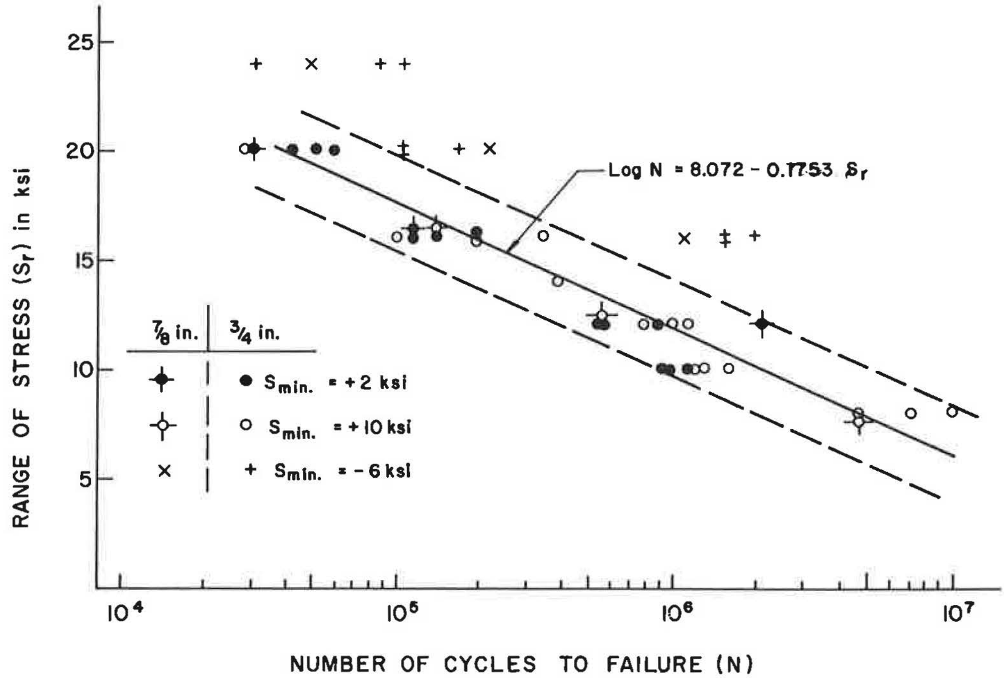


Figure 11. Comparison of push-out test data for ¾-in. and ⅞-in. diameter studs.

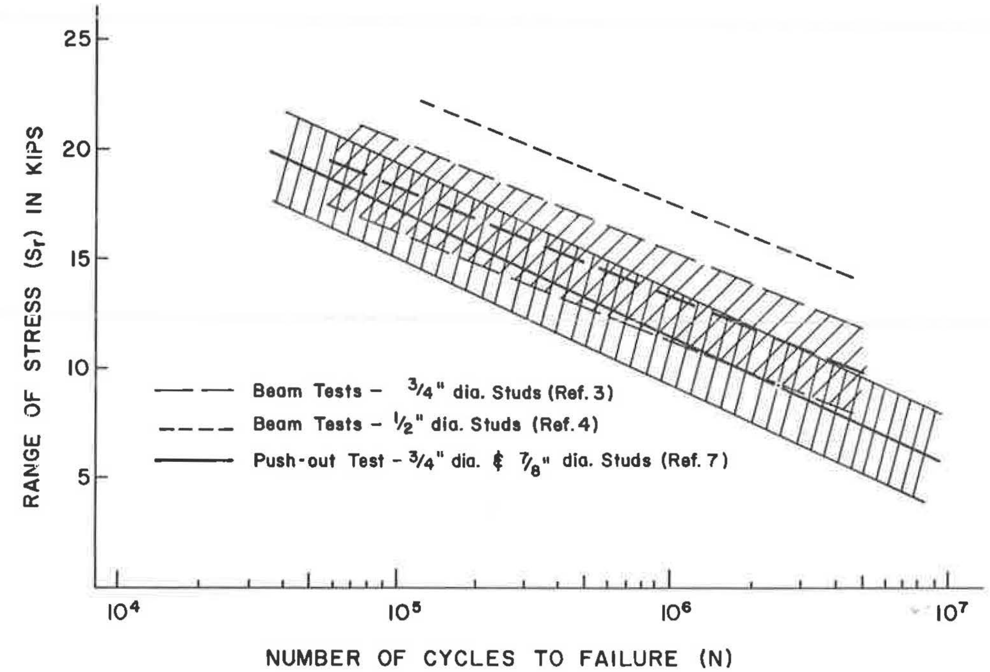


Figure 12. Comparison of S-N curves of beam tests and push-out tests.

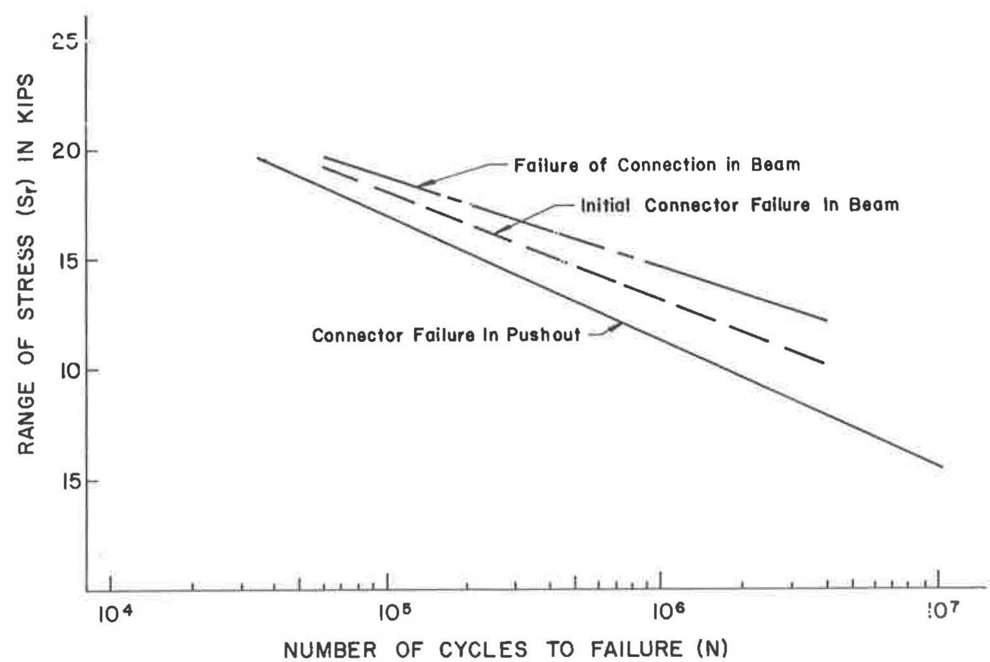


Figure 13. Comparison of initial connector failure and final failure of shear connection in composite beams.

connectors as circles. It is obvious and is verified by analysis that there are no significant differences between the fatigue strengths of ¾-in. and ⅞-in. stud shear connectors.

The test data for the 4-in., 5.4-lb channel shear connectors plotted in Figure 8 also indicate that the stress reversal specimens had significantly greater fatigue strengths than the other 2 levels of minimum stress. Also, it is apparent that the test data for minimum stress levels of 0.5 k/in. and 1.5 k/in. are not significantly different. Hence, as with the stud shear connectors, a conservative estimate of the fatigue strength for all minimum stress levels can be obtained by neglecting the stress reversal data. A regression analysis of the test data for shear loading in only one direction yielded

$$\log N = 8.9470 - 0.8554\, S_r \tag{3}$$

The coefficient of correlation was 0.8648 and the standard error of estimate 0.1975.

Figure 12 compares the regression curve for the push-out specimens reported herein with the beam tests reported earlier (2, 3). It is apparent that the lower limit of dispersion for the beam tests (taken as twice the standard error of estimate) overlaps the upper limit of dispersion for the push-out tests. Hence, the lower limit of dispersion of the beam tests is about equal to the mean behavior of the push-out specimens. This finding is reasonable because in the beam tests a loss of interaction was noted which allowed the connector forces to redistribute and resulted in a less severe stress condition than computed from elastic theory assuming complete interaction. In the push-out specimens the loading on the connectors was maintained at a reasonably constant level throughout the cycle life. Push-out tests therefore represent a lower bound for connector failure.

Also, it should be noted that the failure criteria for the beam test results plotted in Figures 9 and 12 were taken as the initial fatigue fracture of one or more connectors. Other studies (2, 3) have shown clearly that the failure of the first connectors has little effect on the beam response and that considerable additional life was available before the beam failed. This is illustrated in Figure 13 where the curve representing initial fatigue fracture of one or more connectors (plotted in Figs. 9 and 12) is compared with the curve relating the cycle life to failure of the connection in a beam (3). It is apparent that considerably longer cycle life was available before the composite beam failed due to the weakened shear connection.

Because the push-out tests provide a lower bound of fatigue strength it seems satisfactory to consider the mean curve shown in Figure 12 (Eq. 2) as the basis for the design of stud shear connectors. A suitable design value can be obtained from this curve for any desired cycle life. For example, if the expected life is 2 million cycles, the resulting allowable stress range is approximately 10 ksi. This value gives a suitable margin of safety with respect to beam test results.

On the basis of these data and rationale, a tentative design formula for the allowable range of load can be obtained from Eq. 2, with the result:

$$Z_r = \alpha\, d_s^2 \tag{4}$$

where

Z_r = allowable range of shear force per stud in pounds;
d_s = diameter of the stud in inches; and
α = 13,800 for 100,000 cycles
10,600 for 500,000 cycles
7,850 for 2,000,000 cycles.

Equation 4 has been developed from tests of ¾-in. and ⅞-in. stud shear connectors. An examination of Figure 12 indicates that it can be conservatively applied to smaller diameter stud shear connectors.

For the channel shear connectors, the fatigue failure was generally initiated in one of the transverse fillet welds and propagated through the weld. It was apparent that the critical parameter was the stress on the throat of the connecting fillet welds. For

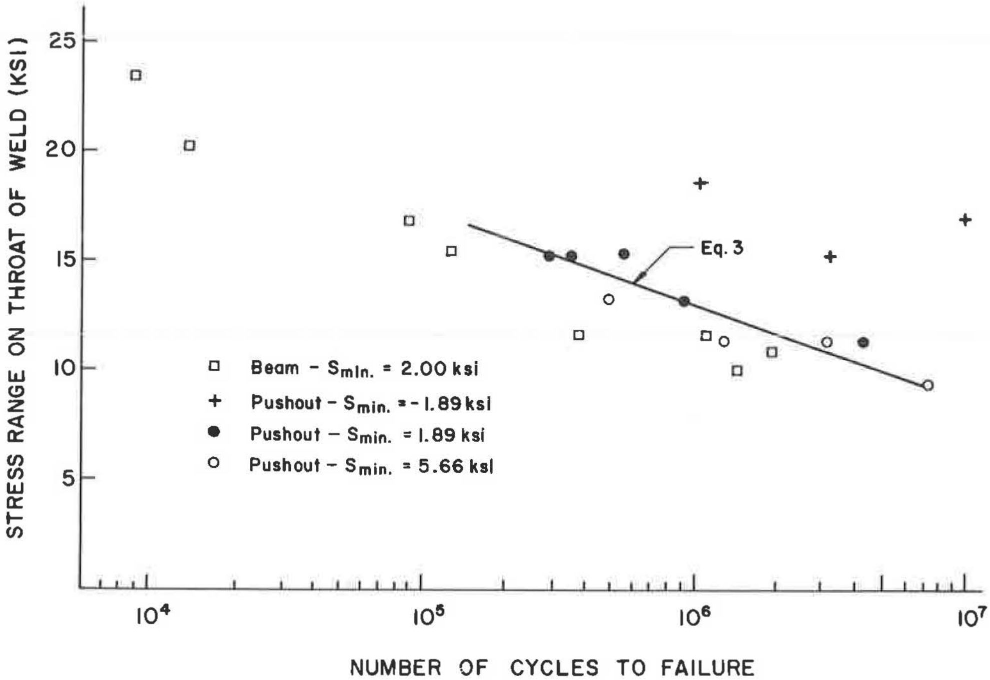


Figure 14. Comparison of Illinois beam test and push-out data for channels.

standard channel sections the thickness of the channel web is always equal to or greater than the thickness at the toe of the channel flange which is uniform at $^3/_{16}$ in. Since the thickness at the toe governs the weld size, it is to be expected that similar behavior should occur in other channels assuming that the same size $^3/_{16}$-in. fillet weld is placed at the heel and toe of the channel.

Equation 3 and the test data for the channel shear connectors are compared with small scale beam tests (6) in Figure 14. The average shear stress range on the throat of the fillet weld is plotted as a function of the logarithm of cycle life. Only those test beams which had shear connectors similar in geometry to the channels in this experiment were considered. Since the size of welds would normally be governed by the flange toe thickness, beams reported (6) with connectors which had the web area reduced so that the size of weld was greater than the web thickness were not considered applicable. In these latter tests the failure plane always occurred in the channel web.

For standard channel sections one should not attempt to place larger welds to provide an increase in the fatigue strength. The beam tests with channel connectors (6) showed that if larger welds were used premature failure would occur in the channel web. Obviously an increase in the fatigue strength of channel connectors could only be achieved if larger welds were used with channels having thicker webs.

Also, it should be noted that the concrete strength varied from about 2500 to 6000 psi in the beam tests. It is apparent that the concrete strength did not significantly influence the fatigue strength of the channel shear connectors which had geometric characteristics similar to the channel used in this experiment. Only when the web area was reduced, or the channel geometry substantially altered from the standard channel geometry, did the strength of concrete influence the cycle life.

It is readily apparent that channel shear connectors can be proportioned from the expected range of shear stress on the throat of the connecting fillet welds, since actual failure was due to fracture of the weld. For convenience, the range of shear in kips per inch of channel width was selected because all standard channels could be expected to behave in a similar manner if $^3/_{16}$-in. fillet welds were placed at the heel and toe.

On this basis, a tentative design formula for the allowable range of load for various cycle lives was developed from Eq. 3 using the lower limit of dispersion. The lower limit of dispersion was used because of the limited amount of test data and the absence of information on full-size beams:

$$Z_r = \beta w \tag{5}$$

where

Z_r = allowable range of shear force in kips per inch;
w = length of a channel shear connector in inches measured in a transverse direction on the flange of a beam; and
β = 4,000 for 100,000 cycles
3,200 for 500,000 cycles
2,600 for 2,000,000 cycles.

DESIGN CRITERIA FOR SHEAR CONNECTORS

It is apparent from the results reported here on the fatigue strength of shear connectors and from recent studies concerned with the ultimate load-carrying capacity of composite members (1) that a different design criterion is needed for the mechanical shear connectors used in composite bridge members. A rational philosophy of design should recognize that adequate static and fatigue strength is required in a bridge structure. Sufficient connectors should be provided to insure the proper fatigue strength. In addition, it is necessary to provide sufficient connectors such that the static ultimate strength of the composite member can be achieved.

Fatigue Considerations

The magnitude of the shear force transmitted by individual connectors has been found to agree closely with values predicted by theory assuming complete interaction within the elastic range (2). Tests have indicated that the difference between the computed values based on complete interaction and the experimental measurements is small. Although these measurements have indicated that connectors in regions of constant shear may not transmit equal forces, the maximum stress on any one shear connector seldom exceeds the value predicted from elastic theory assuming complete interaction.

Since fatigue is critical under repeated applications of working load, it is reasonable to determine the variation in shear stress using elastic theory. In other words the design criterion for fatigue is necessarily based on elastic considerations.

If complete interaction is assumed, the horizontal shear to be transferred by connectors for a given loading can be calculated as

$$H = \frac{VQ}{I} \tag{6}$$

where

H = horizontal shear per inch of length;
V = shear in kips acting on the composite section;
Q = statical moment of the transformed compressive concrete area about the neutral axis of the composite section, in.^{2}; and
I = moment of inertia of the composite section, in.^{4}

In regions of negative moment in continuous beams, the value of Q is the statical moment of the reinforcing steel and the moment of inertia is that of the steel beam and the reinforcing steel. In negative moment regions with continuous reinforcement, flexural conformance and action under working loads produces tensile stresses which are sufficiently large to cause cracking of the slab. Also, with passage of time, shrinkage will occur and hairline cracks will form. The composite bridges at the AASHO Road Test showed that even with large numbers of shear connectors, transverse shrinkage

cracks formed in the slabs of simple beams which allowed passage of water through the slab (9). Hence, it appears reasonable to only consider the cracked section of the concrete slab. It should also be noted that under initial loading when the slab may remain uncracked, in all probability high friction forces can be developed due to bond between the steel beam and the concrete slab. Hence, the connectors would not be required to transmit the greater range of shear alone. After cracking, this frictional force is reduced with continued application of loads as the bond is destroyed.

Placing the shear connectors in the negative moment regions should also assist in maintaining flexural conformance throughout the continuous beam. This also prevents the sudden transition from a composite to a non-composite section when they are omitted. Their placement should minimize the large differentials in deformation that might otherwise occur (as in a coverplated beam) and reduces the danger of fatigue failure in connectors adjacent to the negative moment region.

In simple span beams the range of shear stress throughout the span is dependent on the length of span and the type of loading. For spans up to about 70 ft the range of shear varies from a maximum at the end of the span to about 85 percent of the maximum near midspan. For longer spans this variation is not nearly as great, so that the range of shear is nearly constant throughout the span. This is illustrated by the shear envelopes

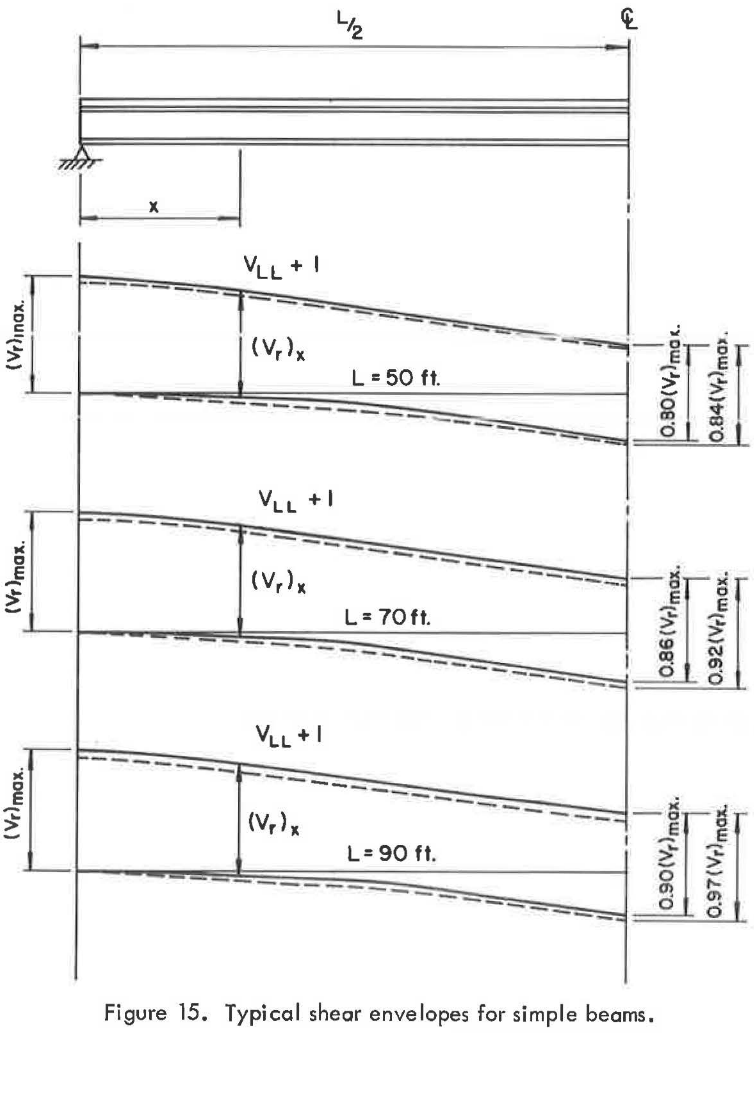


Figure 15. Typical shear envelopes for simple beams.

plotted in Figure 15. At the supports, the horizontal shear computed from Eq. 6 varies from zero to a maximum value as the live load moves onto the span. As is readily apparent from Figure 15, the range of horizontal shear stress will vary from zero to maximum at the supports to near full reversal at midspan. The dashed curves in Figure 15 indicate the maximum shear envelopes for loads moving in the opposite direction. At any section along the span the range of shear is the difference between the maximum and minimum shear envelopes and is indicated in Figure 15 as $(V_r)_x$.

The situation represented by the two outer shear envelopes in Figure 15 (the upper solid curves and the bottom dashed curves) is that of truck loads passing in both directions in the same lane. Although this is not a realistic condition, it is given here for purposes of discussion. The actual envelopes which apply to range of stress on connectors with traffic in one direction are the 2 solid or the 2 dashed curves, depending on the direction which the traffic is moving. The 2 outer shear envelopes could be used to establish a conservative approximation for the stress range throughout the span. For a 50-ft span the resulting range of shear at midspan is approximately 84 percent of the range of shear at the support and for a 90-ft span it is approximately 97 percent of the range of shear at the support. The difference in the resulting range of shear and the actual range of shear is usually less than 5 percent at midspan. This procedure is convenient to use since the actual range of shear is difficult to establish.

For design, an average of the range of shears at the support and at midspan could be used to ascertain the required number of shear connectors where the range of shear is the difference in the minimum and maximum shear envelopes for passage of the vehicle.

An alternate, more conservative, yet simpler procedure would result by considering only the maximum shear at the support. In longer span bridges, the range of shear is more nearly uniform than in the shorter spans so that such an approach would be more conservative for the short span structures. Using the shear at the support as the range of shear throughout the span results in a uniform spacing of the shear connectors. In many structures, the range of shear is nearly uniform and even if the actual shear range were used a nearly uniform spacing would result.

For continuous spans, the variation in the minimum-maximum shear envelopes along the lengths of the spans is usually somewhat greater than in simple spans. Figure 16 shows the moment and shear envelopes for a typical continuous bridge structure. If the variation in the shear stress range is significant, a variable spacing of the connectors is necessary. The range of stress on the connectors in the positive moment regions can be determined in the same manner suggested for simple span structures. The appropriate shear range and the usual composite beam properties of the cross section would be used.

In negative moment regions, the range of horizontal shear acting on the connectors is caused by the force in the reinforcing steel. This shear range can be evaluated from the shear envelopes $(V_r)_x$ and the cross-sectional properties of the beam in that region. As was noted, the value of Q will be the statical moment of the area of reinforcing steel, and the moment of inertia will be that of the steel beam and the reinforcing steel.

For unusual continuous span combinations, positive moments at an interior support may control and more shear connectors may be required to resist the resulting shear. In such instances, the range of shear would vary from zero to a maximum shear associated with the maximum positive moment. This condition only controls with 3- or 4-span continuous beams with odd span ratios such as 10:6:6 or 10:7:7:10.

Since the fatigue investigation reported here has indicated that stress range is the major variable influencing the fatigue strength of the shear connector, sufficient connectors can be provided for any desired cycle life.

The primary design consideration should be based on the fatigue criterion. The range of horizontal shear stresses is computed from Eq. 6. The spacing of the shear connectors is given by

$$P = \frac{\Sigma Z_r}{H_r} \tag{7}$$

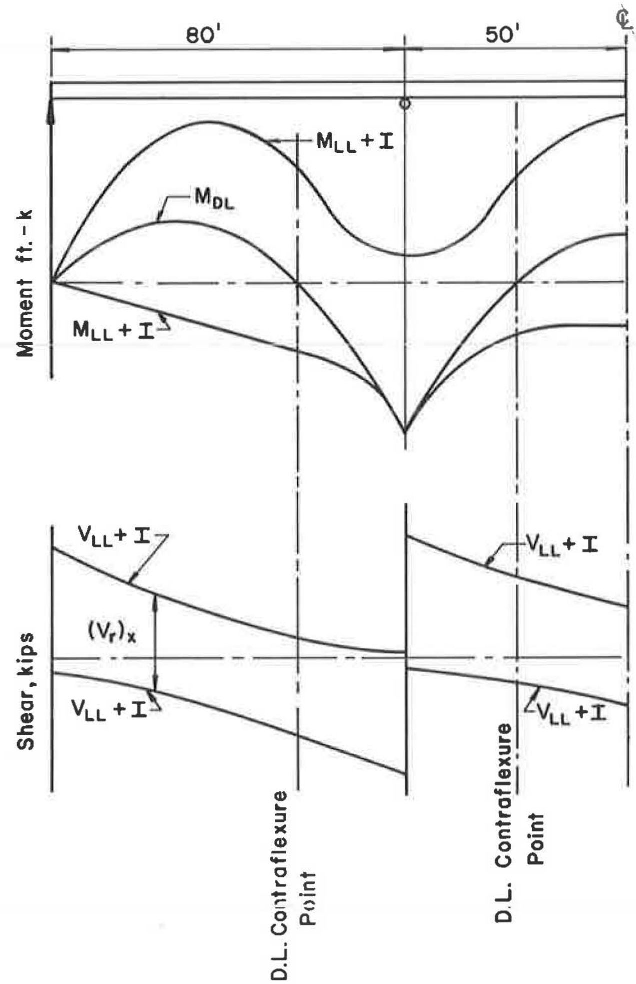


Figure 16. Typical shear and moment envelopes for continuous beams—from Viest et al. (10, p. 95).

where

H_r = the range of horizontal shear per inch of beam length at the junction of the slab and girder. [Note: The quantity H_r is determined from Eq. 6. As is noted in Figures 15 and 16, the range of shear due to live load and impact at any location is $(V_r)_x$.]

Z_r = allowable range of horizontal shear for the connector evaluated from Eqs. 4 or 5 (ΣZ_r is the resistance of all connectors at one transverse cross section of the girder).

P = spacing of shear connectors.

Equation 7 will determine the spacing in most designs. The spacing of connectors should never exceed 24 in. because connectors also perform the necessary function of holding the concrete slab in contact with the steel beam.

Flexural Strength Requirements

In addition to providing adequate fatigue strength, sufficient connectors should be provided to insure that the flexural strength of the composite member can be reached. Usually this requirement will be satisfied in most composite beams because fatigue considerations are usually critical except in cases of shored construction.

Recent research has shown that the flexural strength of composite beams can be achieved if sufficient connectors are provided to resist the maximum horizontal force in the slab (1). This study also confirmed that connector spacing was not critical and that connectors could be spaced uniformly without deleterious effects on the ultimate strength.

At the ultimate moment of a composite beam, 2 stress distributions are possible, as shown in Figure 17. Other studies (1) have demonstrated that the horizontal force required for the determination of the number of shear connectors is the compressive force in the concrete slab when the fully plastic stress distribution for the ultimate flexural strength is reached. For the two cases possible (Fig. 17), the maximum horizontal force is given by

$$H_1 = A_s F_y \tag{8}$$

$$H_2 = 0.85 f'_c bc \tag{9}$$

where

A_s = total area of the steel section including coverplates;
F_y = minimum yield point of the type of steel being used;
f'_c = compressive strength of concrete at 28 days;
b = effective width of the concrete slab; and
c = thickness of the concrete slab.

For any composite section the ultimate flexural strength will be governed by either Eq. 8 or Eq. 9. When the slab is large compared with the beam section, the yield strength of the steel section governs (Eq. 8). When the beam section is large compared with the slab, the ultimate compressive strength of the slab governs (Eq. 9). Obviously for any given composite beam the maximum possible compressive force in the concrete slab would necessarily have to be the smaller of the 2 values computed from Eqs. 8 and 9. Hence, between a point of maximum positive moment and the end of a beam or point of dead load contraflexure, sufficient connectors should be provided to resist the smaller value given by Eqs. 8 and 9.

For continuous beams an additional force in the slab between a point of dead load contraflexure and an adjacent interior support must be resisted, as indicated in Figure 18.

As the plastic moments of the continuous beam are approached and hinges develop, extensive cracks form over the supports of continuous composite beams. Therefore, in continuous beams the portion of the beam between a point of contraflexure and a point of maximum negative moment must be provided with sufficient shear connectors to resist the horizontal force, H_3, equal to the yield strength of the longitudinal reinforcement of the slab:

$$H_3 = A_s^r F_y^r \tag{10}$$

where

A_s^r = total area of reinforcing steel in the slab at the interior support, and

F_y^r = minimum yield point of reinforcing steel.

It has been shown (1) that the ultimate strength of shear connectors is given by the expressions

$$\text{Stud connectors} \qquad q_u = 930\, d_s^2 \sqrt{f'_c} \tag{11}$$

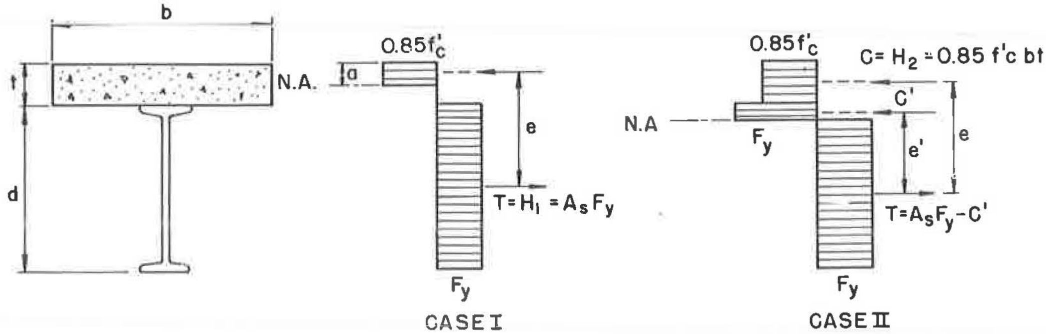


Figure 17. Stress distribution at ultimate load.

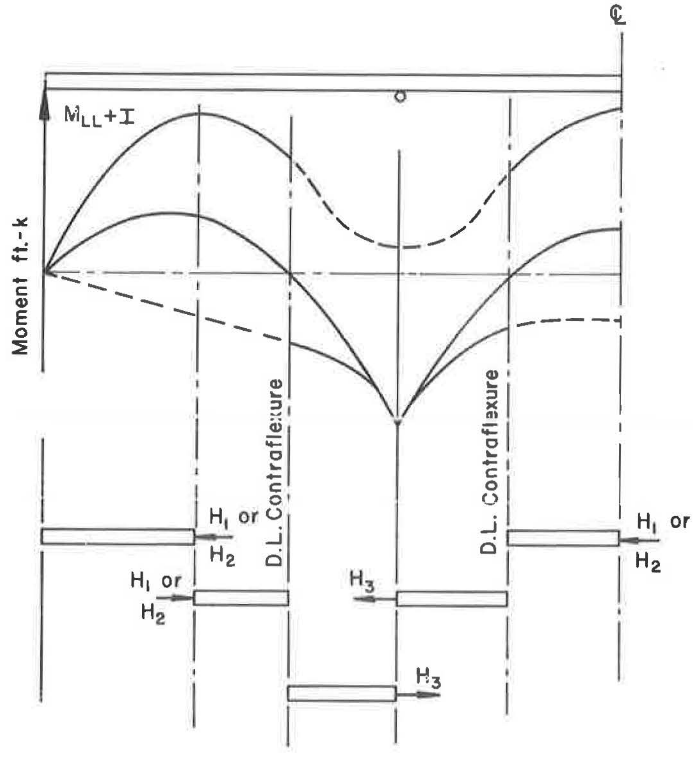


Figure 18. Forces acting on the slab of continuous beams.

Channel connectors $$q = 550\,(h + 0.5t)\,w\sqrt{f'_c} \quad (12)$$

To insure the development of the ultimate flexural strength of composite beams, a larger margin of safety against connector failure should be provided than is provided for the beam. Historically, the factor of safety for connections and fasteners has been larger than for the connected members. This assures that the connections do not fail before the main members. This margin can be accomplished by providing a load reduction factor (ϕ) to the ultimate shear strength of the shear connector. A ϕ value of

0.85 appears to be reasonable. Since the ultimate flexural capacity of composite bridge beams is usually 2.5 or more times the working load moment, the corresponding margin for the shear connection would be approximately 3 or greater.

Tests reported earlier (1) demonstrated that only a slight deformation in the concrete near the more heavily stressed connectors is needed to redistribute the horizontal shear to other less heavily stressed connectors. A few tests were reported by Culver and Coston (8) in which the loading positions for 2-point loading were changed in successive tests by moving the loads toward the supports. These tests have indicated that the maximum load can most likely move into the position of maximum moment without premature failure in a more heavily stressed connector because of these redistribution characteristics.

It should be noted that seldom will the maximum load criterion be the governing factor. The number of connectors required by the fatigue criterion will usually exceed the requirements for ultimate flexural strength.

In instances where the maximum load criterion represents the more critical condition, the use of the flexural strength requirements insures that sufficient connectors are present so that excessive local permanent deformation of the concrete in the vicinity of stud connectors is minimized. If this provision was not used local deterioration of the concrete could result which would adversely influence the fatigue strength of stud connectors. Channel shear connectors have sufficient bearing area so that it is doubtful that excessive local permanent deformation would occur.

In simple beams the stress range values for stud and channel connectors will govern the design and it would not ordinarily be necessary to calculate both values since the fatigue criterion is the more critical condition. This is also true in continuous beams for unshored construction.

The minimum number of shear connectors required between the points of maximum positive moment and the end supports or dead load points of contraflexure, and between points of maximum negative moment and the dead load points of contraflexure is given by

$$N_i = \frac{H_i}{\phi q_u} \tag{13}$$

where

N_i = the minimum number of shear connectors between points of maximum positive moment and adjacent end supports or dead load points of contraflexure or between points of maximum negative moment and adjacent dead load points of contraflexure.

ϕ = a reduction factor = 0.85.

q_u = ultimate shear connector loads given by Eqs. 11 or 12.

H_i = smaller value of H_1 and H_2 (Eqs. 8 or 9) for: simple beams; continuous beams between points of maximum positive moment and the end supports; and continuous beams between points of maximum positive moment and points of dead load contraflexure.

= H_3 (Eq. 10) for continuous beams between points of dead load contraflexure and interior supports.

If the number of shear connectors given by Eq. 13 exceeds the number provided by the spacing given by Eq. 7, additional connectors should be added to insure that the ultimate strength is achieved.

SUMMARY AND CONCLUSIONS

1. Tests of 35 push-out specimens having the concrete slab connected to the steel beam section with ¾-in. stud shear connectors, 9 tests with ⅞-in. stud connectors, and 12 tests with 4-in., 5.4-lb channel connectors were made to determine the fatigue behavior of the connectors.

2. A mathematical model expressing the logarithm of the fatigue life as a linear function of stress range was found to fit the test data. An analysis of variance indicated that minimum stress was a significant variable only for stress reversal. If the reversal portion was neglected the stress range was by far the most important independent variable.

3. Concrete strength did not significantly influence the fatigue strength of either the stud or the channel shear connector.

4. The push-out specimen developed for this study provided test results which overlap the earlier beam tests. The lower limit of dispersion for the beam tests overlaps the upper limit of dispersion for the push-out test. Hence, the lower limit of dispersion of the beam tests is about equal to the mean behavior of the push-out specimens. Push-out tests therefore represent a lower bound for connector failure.

5. A design criterion for shear connectors is proposed which recognizes both the static and fatigue behavior of the shear connectors for composite steel and concrete members.

6. A design procedure is developed which provides a simpler and more economical design for the shear connectors of composite beams.

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